PORTLAND HARBOR RI/FS

APPENDIX C

TECHNOLOGY ASSIGNMENT SUPPORTING DOCUMENTATION

FEASIBILITY STUDY

DRAFT FINAL

July 29, 2015

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C1.0 INTRODUCTION

This appendix presents supporting information for the technology assignment process described in Section 3 of the Portland Harbor Superfund Site (Site) Feasibility Study (FS). The technology assignment process combines decision tree and multi-criteria decision matrix components (FS Section 3.3.2) and uses a GIS-based tool to score the various technologies. This appendix provides supporting information on the sources of information used for each decision tree and matrix criterion used within the GIS-based tool. Information is provided for the following criteria:

Decision Tree Criteria

- Navigation Channel and Future Maintenance Dredge Areas
- Final Remedy Areas

Multi-Criteria Decision Matrix

- Hydrodynamic Characteristics
    - Sediment Deposition Rate
        - Deposition Based on Bathymetric Surveys
        - Ratio of Subsurface to Surface Sediment Concentrations
    - Sediment Erosion Potential
        - Wind and Wake Generated Waves
        - Shear-Stress on Bottom Sediments
    - Shallow Water Depth
- Sediment Bed Characteristics
    - Sediment Slope
- Anthropogenic Influences
    - Structures and Pilings
    - Debris
    - Propwash

C2.0 DECISION TREE CRITERIA

The sources of information used to define the navigation channel, future maintenance dredge areas, and final remedy areas are provided below.

C2.1 NAVIGATION CHANNEL AND FUTURE MAINTENANCE DREDGE AREAS

Congress authorized the federal navigation project within the Willamette River and defined the boundaries of the federal navigation channel. A GIS layer used to define the navigation channel and future maintenance dredge areas was developed by the LWG and provided to EPA in May 2012.

Future maintenance dredge areas were identified through a site use survey distributed to LWG members in November 2008 to gather information on existing and future activities at various locations along the Superfund Site to inform FS site use assumptions. Topics addressed in the survey included vessel activity, number and type of dock structures, shoreline characteristics, outfall locations, potential restoration areas, and potential future development or in-water construction. Information obtained from the survey related to dock configuration and future site uses was used to develop estimates of likely future navigation depth requirements and potential future maintenance dredging depths near and around docks.

Commented [RC1]: We suggest describing this survey in more detail, as many of these topics likely required some research. For instance, were survey respondents required to submit documentation supporting their estimates? Were these values based on one person's response to the survey or was it a consensus from several people at the company? Alternatively, if there is a publically-available report describing this survey, that report could be referenced.

C2.2 FINAL REMEDY AREAS

Only one final remedy area, the McCormick and Baxter cap, is located within the Site. The cap was placed over contaminated sediments in September 2005; subsequent modifications were made to the cap in October 2005 and July 2007. The cap design incorporated different types of armoring in the nearshore areas to reduce erosion (DEQ 2005). The GIS layer identifying the final remedy area at the McCormick and Baxter site was provided by LWG as part of their "Dredge/Cap Areas" GIS layer.

Commented [RC2]: For those interested in seeing the final remedy location, we suggest including a reference to a particular figure that was made using this layer.

C2.3 HYDRODYNAMIC CHARACTERISTICS

The sources of information used to define sediment deposition rates, sediment erosion potential, and shallow water depths are provided below.

C2.3.1 Sediment Deposition Rate

Sediment deposition rate was evaluated based on two lines of evidence: 1) quantitative evaluation of the difference between bathymetric surveys conducted at the site, and 2) the ratio of subsurface to surface sediment concentrations, which assumes that depositional processes have led to cleaner sediments overlying more contaminated sediments.

Commented [RC3]: We suggest mentioning that these were bathymetric surveys conducted at the site in different years.

Commented [RC4]: We suggest indicating that these topics will be discussed below. As it is, I was expecting more information relating to the assumption in number two.

C2.3.2 Deposition Based on Bathymetric Surveys

Sediment deposition or erosion has been measured empirically at the Site through bathymetric surveys conducted in 2002/2003 and January 2009. Based on the accuracy of the surveys (+/- 0.5 feet) and the time frame being considered (7 years or 5.67 years depending on whether the January 2002 or May 2003 is selected as the initial survey date), the minimum detectable sediment deposition rate was estimated to range between 2.2 and 2.7 centimeters per year (cm/yr). Thus, a sediment deposition rate of 2.5 cm/year was identified as the threshold for establishing an area as depositional based on this line of evidence. Areas with deposition greater than 2.5 cm/yr received a value of 1

Commented [RC5]: It would be better to say "…through three bathymetric surveys conducted…". As it is currently written, it looks like two surveys had been conducted – one of which spanned two years.

Commented [RC6]: We do not understand why 2.5 cm/yr would be identified as depositional if 2.7 cm/yr was the minimum detectable sediment deposition rate for one of the study year comparisons. It seems as though if values are equal to or greater than 2.7 cm/yr (essentially the sediment deposition detection limit) then the area should receive a value of 1.

This would cause more areas to be classified as erosional in the Willamette River and may influence the selected remedy.

We request additional justification for this decision and/or a change to the analysis assumptions.

(indicating a depositional environment) while other areas received a 0 when constructing the technology assignment GIS layer. This information was used in the final depositional criteria process.

C2.3.3 Ratio of Subsurface to Surface Sediment Concentrations

The ratio of subsurface to surface sediment concentrations was determined by calculating the average subsurface (greater than 40 cm depth) and surface sediment concentrations for PCBs, PCDD/PCDFs, PAHs, and DDx. GIS rasters were developed using a natural neighbor interpolation of surface and subsurface sediment concentrations. Subsurface rasters were divided by the corresponding surface raster for each of the focused COCs, and were then mosaicked to a new raster layer using the mean operator. The resulting raster was reclassified to identify all areas where the ratio was greater than two. Areas where the ratio was greater than 2 were assigned a value of 1 indicating a depositional environment. Areas where the ratio was less than 2 were assigned a value of 0. This information was used in the final depositional criteria process. Where the concentration in surface sediment was less than the chemical-specific G-RAL, surface-subsurface concentration ratios were not calculated, as those locations are outside the boundary of proposed cap/dredge areas.

Commented [RC7]: The comparison of subsurface concentrations to surface concentrations includes no discussion of other processes that may be at play. For instance, degradation rates of contaminants are often different in subsurface sediment conditions as compared to surface sediment conditions. It is unclear how much this would affect the ratio, and therefore it is unclear if the ratio really provides a complete picture of deposition.

We suggest providing a discussion of degradation rates of these chemicals in the surface and subsurface to either 1) show that the degradation rates are equivalent, or 2) provide information on the effect that the difference in these rates would have on the analysis.

C2.4 SEDIMENT EROSION POTENTIAL

Two lines of evidence were used to indicate whether an area was erosive; wind and vessel wake generated waves, and shear-stress on bottom sediments during high flow events.

C2.4.1 Wind and Wake Generated Waves

LWG conducted a wave analysis using information on waterway traffic obtained from the U.S. Army Corps of Engineers (USACE), Port of Portland, and correspondence with other property owners. The analysis considered both wind-generated (wave) and vessel-generated (wake) wave heights at variable river stage elevations to define wind and wake generated wave zones and derive a GIS layer.

Surface waves generated by wind conditions and vessel activity adjacent to each SDU were estimated. Evaluation of wind-induced waves included meteorological data acquisition and wave hindcasting to develop significant wave heights and peak wave periods. Evaluation of vessel-induced waves included research of vessel traffic and vessel-wake generation to develop wake heights produced by design vessels operating at various speeds and water depths.

Commented [RC8]: It is unclear whether the surface wave heights were estimated or if the wave zone was estimated. We suggest clarifying this.

Commented [RC9]: Was this primary, on-the-water research or research involving publicly available documents? I believe this is explained later, but it would be worth noting “(described below)” for clarity.

Design Water Levels

Water levels in the lower reach of the Willamette River exhibit an average 2-foot fluctuation due to tidal influence. They are also affected by the stage in the Columbia River, which is regulated by the Bonneville Dam upstream, and by runoff during extreme rainfall-runoff events. LWG obtained the U.S. Geological Survey (USGS) maximum and minimum daily stage river data (USGS gage 14211720, Willamette River at Portland,

Commented [RC10]: We suggest including a note at the end of this sentence, for context, reporting what the maximum and minimum fluctuation is.

Also, is this average based on values measured on days not influenced by storm activity (i.e., this is truly only tidally influenced fluctuation)? If days with storm activity are included, then we suggest revising this statement to say so.

Oregon), and the maximum and minimum extreme stage data from the USACE for the 1973 to 2003 period (USACE 2004). The USACE defined the ordinary high water mark (OHW) at 19.8 feet North American Vertical Datum of 1988 (NAVD88) (14.8 feet Columbia River datum[CRD]). The minimum extreme stage in the river was estimated at 4.5 feet NAVD88 (-0.5 feet CRD). LWG limited the study to the river water levels between minimum extreme stage (4.5 feet NAVD88) and 13 feet NAVD88.

Evaluation of Wind-Induced Waves
Wind-generated waves are anticipated to be small compared to vessel-generated wakes along the Site. This is primarily due to the short fetch distances (distance over water that the wind can blow without being impeded by land) at the Site, which will limit the size of wind-generated waves that can develop in the lower Willamette River. To a lesser extent, the sinuosity of the lower Willamette River also limits wind-generated wave growth and propagation by limiting the straight line distances along which waves can develop and propagate. The methodology and results for the wind-induced wave evaluation are described below:

Wind Data Sources and Pre-processing
Wind data were obtained for the Portland International Airport from the National Climatic Data Center (http://www.ncdc.noaa.gov/oa/ncdc.html; 1976 to 2004) and the Meteorological Resource Center (http://www.webmet.com/; 1961 to 1990). Data were compiled into a single set and wind speeds were adjusted to two-minute averages at a 10-meter above ground elevation for analysis using methodology outlined in the USACE Coastal Engineering Manual (CEM) (USACE 2002). The use of 2-minute averages was chosen to provide a conservative estimate of wind-generated wave heights. **Figure C-1** illustrates a wind rose of the combined dataset. Dominant wind directions at these locations, as shown in **Figure C-1**, are from the northwest and southeast.

100-Year Return Period Wind Speeds
Twelve wind direction zones were defined, each encompassing a 30° range starting from 0°N. For each zone, the annual maximum wind speed for each year from 1961 to 2004 with a direction falling within the zone was identified. A Rayleigh distribution curve was fitted to the annual maxima data and the 100-year return period wind speed was extrapolated for each directional zone. This distribution produced a good fit to the wind dataset with correlation coefficients ranging from 0.84 to 0.98, with an average of 0.94. **Table C-1** outlines the 100-year wind speed and Rayleigh correlation coefficient for each directional bin.

Table C-1. 100-year Return Period Wind Speeds

Directional Zone (°N)	100-year Wind Speed (mph)	Rayleigh Correlation Coefficient (R^2)
0 to 30	30	0.95
31 to 60	37	0.96
61 to 90	56	0.97
91 to 120	59	0.97
121 to 150	40	0.97
151 to 180	59	0.98
181 to 210	69	0.84
211 to 240	60	0.89
241 to 270	47	0.97
271 to 300	39	0.96
301 to 330	38	0.95
331 to 360	37	0.97

Fetch Length Determination

Fetch lengths were measured for each wind directional zone that has the potential for wind waves to develop and impact the shoreline for the SDUs and other areas of the Site. Fetch measurements were completed based on methodology outlined in the CEM (USACE 2002). These fetch lengths and associated directions are listed in **Table C-2**.

Table C-2. Fetch Lengths (in feet) and Associated Wind Parameters for Various SDUs

Start Heading (°N)		0	31	61	91	121	151	181	211	241	271	301	331
End Heading (°N)		**30**	**60**	**90**	**120**	**150**	**180**	**210**	**240**	**270**	**300**	**330**	**360**
100-year Wind Speed (mph)		**30**	**37**	**56**	**59**	**40**	**59**	**69**	**60**	**47**	**39**	**38**	**37**
SDU	RM2E								4,400	2,100	2,100		3,400
	RM3.5E							3,700	1,900	1,900	4,600	4,300	
	RM3.5E						4,600	3,500	1,900	1,600	2,400	4,600	
	RM3.9W	3,100	1,800	1,900	2,400	4,200							
	RM3.9W	3,300	1,800	2,100	2,700	4,700							
	RM4.5E						4,600	3,000	1,900	2,000	2,700	5,300	
	RM5W	3,200	2,000	1,600	2,800								5,400
	RM5W	2,500	1,400	1,400	2,600								4,300
	RM6W	1,600	1,200	1,300	3,500								3,800
	RM6W	1,400	1,300	3,300	5,000							3,000	2,600
	RM5.5E						3,100	1,700	1,200	1,500	2,700	4,200	
	RM5.5E						2,600	1,800	1,200	1,400	3,200		
	RM6.5E					2,700	2,200	1,400	1,600	1,600	3,700		
	RM6.5E					3,800	3,800	1,500	1,500	2,400	2,900		
	RM7W	2,400	2,000	2,200	3,500								3,000
	RM6.5E							1,900	2,000	2,600	6,200		
	RM7W	3,200	2,700	4,200	3,800								3,800
	Swan Is						4,000	2,500	2,500	2,200	3,300		
	Swan Is								3,900	5,400			
	RM9W	2,800	1,700	2,600	3,500								4,600
	RM9W	2,100	1,900	3,100	4,400							5,900	5,900
	RM9W	1,800	1,800	3,100	3,200							3,600	3,100
	RM9E*					3,800	2,800	1,700	1,900	3,500	4,300		
	RM9E*					4,300	2,900	2,000	1,600	2,800	3,700		
	RM10E*						2,100	1,900	1,500	2,900	5,000		
	RM10W*	1,700	1,100	1,200	2,800								3,300
	RM11E					2,800	2,500	1,300	1,200	1,500	2,500		
	RM11W*	1,300	1,100	1,700	2,300								3,200

Notes:
* These are not official SDUs but represent approximate locations at the Site.

Estimates of Wind-Generated Wave Heights/Periods

The 100-year return period wave heights and periods for each relevant directional zone were calculated based on the restricted-fetch wave growth formulation in the Automated Coastal Engineering System (ACES) developed by the USACE (1992). **Tables C-3** and **C-4** present the 100-year significant wave heights and periods, respectively, for each directionally applicable combination of 100-year wind speed and fetch length. **Table C-5** outlines the maximum significant wave heights and periods developed for each SDU or area of the Site. Maximum 100-year significant wave heights estimated at each area ranged from 1.4 feet to 2.2 feet. Associated wave periods ranged from 2.0 to 2.5 seconds. The variation in 100-year significant wave height along the project reach is estimated to be only about 0.8 feet; therefore, the design wind-generated significant wave height and period for evaluation of shoreline armoring along the entire project reach is defined as 2.2 feet and 2.5 seconds, respectively.

Table C-3. 100-year Significant Wave Heights (in feet) and Associated Wind Parameters for Each SDU

	Start Heading (°N)	**0**	**31**	**61**	**91**	**121**	**151**	**181**	**211**	**241**	**271**	**301**	**331**
	End Heading (°N)	**30**	**60**	**90**	**120**	**150**	**180**	**210**	**240**	**270**	**300**	**330**	**360**
	100-year Wind Speed (mph)	**30**	**37**	**56**	**59**	**40**	**59**	**69**	**60**	**47**	**39**	**38**	**37**
SDU	RM2E								2.0	1.0	0.8		1.0
	RM3.5E							2.2	1.3	1.0	1.2	1.1	
	RM3.5E						2.0	2.1	1.3	0.9	0.9	1.2	
	RM3.9W	0.7	0.7	1.2	1.5	1.2							
	RM3.9W	0.8	0.7	1.3	1.6	1.3							
	RM4.5E						2.0	2.0	1.3	1.0	0.9	1.3	
	RM5W	0.7	0.8	1.1	1.6								1.2
	RM5W	0.7	0.6	1.1	1.5								1.1
	RM6W	0.5	0.6	1.0	1.8								1.0
	RM6W	0.5	0.6	1.6	2.1							1.0	0.9
	RM5.5E						1.7	1.5	1.1	0.9	0.9	1.1	
	RM5.5E						1.5	1.5	1.1	0.9	1.0		
	RM6.5E					1.0	1.4	1.4	1.2	0.9	1.1		
	RM6.5E					1.1	1.8	1.4	1.2	1.1	1.0		
	RM7W	0.6	0.8	1.3	1.8								0.9
	RM6.5E							1.6	1.4	1.2	1.4		
	RM7W	0.7	0.9	1.8	1.8								1.0
	Swan Is						1.9	1.8	1.5	1.1	1.0		
	Swan Is								1.9	1.7			

	Start Heading (°N)	0	31	61	91	121	151	181	211	241	271	301	331
	End Heading (°N)	**30**	**60**	**90**	**120**	**150**	**180**	**210**	**240**	**270**	**300**	**330**	**360**
	100-year Wind Speed (mph)	**30**	**37**	**56**	**59**	**40**	**59**	**69**	**60**	**47**	**39**	**38**	**37**
	RM9W	0.7	0.7	1.4	1.8								1.1
	RM9W	0.6	0.7	1.6	2.0							1.3	1.3
	RM9W	0.6	0.7	1.6	1.7							1.0	0.9
	RM9E*					1.1	1.6	1.5	1.3	1.3	1.2		
	RM9E*					1.2	1.6	1.6	1.2	1.2	1.1		
	RM10E*						1.4	1.6	1.2	1.2	1.3		
	RM10W*	0.5	0.6	1.0	1.6								1.0
	RM11E					1.0	1.5	1.3	1.1	0.9	0.9		
	RM11W*	0.5	0.6	1.2	1.4								1.0

Notes:
* These are not official SDUs but represent approximate locations at the Site.

Table C-4. 100-year Significant Wave Periods (in sec) and Associated Wind Parameters for Each SDU

	Start Heading (°N)	0	31	61	91	121	151	181	211	241	271	301	331
	End Heading (°N)	**30**	**60**	**90**	**120**	**150**	**180**	**210**	**240**	**270**	**300**	**330**	**360**
	100-year Wind Speed (mph)	**30**	**37**	**56**	**59**	**40**	**59**	**69**	**60**	**47**	**39**	**38**	**37**
SDU	RM2E								2.5	1.8	1.6		1.8
	RM3.5E							2.6	2.0	1.8	2.0	2.0	
	RM3.5E						2.5	2.6	2.0	1.7	1.7	2.0	
	RM3.9W	1.6	1.5	1.9	2.1	2.0							
	RM3.9W	1.6	1.5	2.0	2.2	2.1							
	RM4.5E						2.5	2.5	2.0	1.8	1.8	2.1	
	RM5W	1.6	1.6	1.8	2.2								2.1
	RM5W	1.5	1.4	1.8	2.2								1.9
	RM6W	1.3	1.4	1.7	2.3								1.9
	RM6W	1.3	1.4	2.2	2.6							1.8	1.7
	RM5.5E						2.3	2.1	1.8	1.7	1.8	1.9	
	RM5.5E						2.2	2.1	1.8	1.6	1.8		
	RM6.5E					1.8	2.1	2.0	1.9	1.7	1.9		
	RM6.5E					1.9	2.4	2.0	1.9	1.9	1.8		

Start Heading (°N)	**0**	**31**	**61**	**91**	**121**	**151**	**181**	**211**	**241**	**271**	**301**	**331**
End Heading (°N)	**30**	**60**	**90**	**120**	**150**	**180**	**210**	**240**	**270**	**300**	**330**	**360**
100-year Wind Speed (mph)	**30**	**37**	**56**	**59**	**40**	**59**	**69**	**60**	**47**	**39**	**38**	**37**
RM7W	1.5	1.6	2.0	2.3								1.8
RM6.5E							2.2	2.0	1.9	2.2		
RM7W	1.6	1.7	2.4	2.4								1.9
Swan Is						2.4	2.3	2.2	1.8	1.9		
Swan Is								2.4	2.3			
RM9W	1.5	1.5	2.1	2.3								2.0
RM9W	1.4	1.6	2.2	2.5							2.1	2.1
RM9W	1.4	1.5	2.2	2.3							1.9	1.8
RM9E*					1.9	2.2	2.1	2.0	2.1	2.0		
RM9E*					2.0	2.2	2.2	1.9	2.0	1.9		
RM10E*						2.0	2.2	1.9	2.0	2.1		
RM10W*	1.3	1.3	1.7	2.2								1.8
RM11E					1.8	2.1	2.0	1.8	1.7	1.7		
RM11W*	1.2	1.3	1.9	2.1								1.8

Notes:
* These are not official SDUs but represent approximate locations at the Site.

Table C-5. Maximum 100-year Wind Wave Heights and Periods for Each SDU

SDU	Significant Wave Height (ft)	Significant Wave Period (s)
RM2E	2.0	2.5
RM3.5E	2.1	2.6
RM3.5E	1.5	2.1
RM3.9W	1.6	2.2
RM3.9W	2.0	2.5
RM4.5E	1.6	2.2
RM5W	1.5	2.2
RM5W	1.8	2.3
RM6W	2.1	2.6
RM6W	1.7	2.3
RM5.5E	1.5	2.2
RM5.5E	1.4	2.1
RM6.5E	1.8	2.4
RM6.5E	1.8	2.3
RM7W	1.6	2.2
RM6.5E	1.8	2.4

SDU	Significant Wave Height (ft)	Significant Wave Period (s)
RM7W	1.9	2.4
Swan Is	1.9	2.4
Swan Is	1.8	2.3
RM9W	2.0	2.5
RM9W	1.7	2.3
RM9W	1.6	2.2
RM9E*	1.6	2.2
RM9E*	1.6	2.2
RM10E*	1.6	2.2
RM10W*	1.5	2.1
RM11E	1.4	2.1
RM11W*	2.0	2.5

Notes:
* These are not official SDUs but represent approximate locations at the Site.

Evaluation of Vessel-Generated Waves
Estimates of vessel-induced wave heights were completed through an evaluation of ship traffic patterns within the Site and analytical calculations of vessel wakes based on type of vessel, operational speed, and water depths.

Information on waterway traffic at the Site was obtained from the following sources:

- USACE website database on annual trips and drafts of vessels on the lower Willamette River (USACE 2006)
- USACE website database on vessels residing in the Port of Portland (USACE 2007)
- Port of Portland documentation on arrivals and departures of all industrial vessels in 2008 (Port of Portland 2009)
- LWG property owner Site Use Survey
- Other sources, including correspondence with Foss Maritime Company and Portland Spirit

Commercial vessel traffic between Terminal 2 (RM 10) and Terminal 4 (RM 4.5) was used as representative of commercial vessel operations at the Site within the Willamette River. Commercial vessels operating in this area range from larger cargo vessels and tankers with drafts of less than 40 feet, to smaller push-boats, tugboats, and passenger ships/ferryboats with drafts of less than 18 feet. Overall, 51 percent of commercial vessel traffic consists of tugboats, tows, and push-boats; 44 percent consists of cargo ships; and only 5 percent consists of tankers. Excursion jet boats operated by the Portland Spirit and Willamette Jetboat Excursions travel through the Site several times daily during the summer season (approximately April through September). No available count was found for smaller recreational boats; however, wakes from these vessels are expected to be small compared to those produced by commercial vessels and excursion jet boats.

Estimates of Wakes from Commercial Vessels

The Weggel-Sorensen model (Weggel and Sorensen 1986) calculates wave height generated at a vessel bow as a function of the vessel speed, distance from the sailing line, water depth, vessel displacement volume, and vessel hull geometry (vessel length, beam, and draft). This method has been widely accepted and used for calculating vessel wakes from commercial vessels. Model inputs include water depth, vessel displacement, distance from the sailing line, vessel speed, and bow geometry (or hull form) coefficients. The model results include the wave height and period for the selected distance from the sailing line. The model was applied for all commercial vessels (except for high-speed excursion jet boats, covered in the following section). The results of these calculations for all design conditions are provided in **Attachment C-1**. **Table C-6** lists the maximum wake height calculated for each area studied.

Table C-6. Maximum Wake from Commercial Vessel Traffic (Traveling at Reasonable Speeds) for Each SDU

SDU	Vessel	Wake Height (feet)	Wake Period (sec)
RM2E	Pushboat	2.0	2.7
RM3.5E	Passenger Ferry	2.8	2.7
RM3.9W, RM4.5E, RM5W, RM5.5E	Passenger Ferry	2.8	2.7
RM6W	Passenger Ferry	2.8	2.7
RM5.5E, RM6.5E	Fireboat	2.1	4.0
RM6.5E	Pushboat	2.0	2.7
RM7W	Passenger Ferry	2.8	2.7
RM6.5E	Pushboat	2.0	2.7
RM7W	Passenger Ferry	2.8	2.7
Swan Is, RM9W	No Wake	n/a	n/a
RM9W	Passenger Ferry	2.8	2.7
RM9W	Passenger Ferry	2.7	2.7
RM9W	Pushboat	1.7	2.7
RM9E*	Pushboat	1.7	2.7
RM9E*, RM10E*	Fireboat	2.1	4.0
RM10W*	Passenger Ferry	2.8	2.7
RM11E	Passenger Ferry	2.8	2.7
RM11W*	Passenger Ferry	2.8	2.7

Notes:
* These are not official SDUs but represent approximate locations at the Site.

Commented [RC11]: We suggest stating more explicitly what these speeds are, as "reasonable" is very subjective. For instance, it would make sense if the speeds used were the posted speed limits in the waterway or based on some sort of survey of the commercial boat captains.

Maximum wake heights within the SDUs were due to one of three design vessels (pushboat, passenger ferry, or fireboat) at relatively high speeds. Estimated wake heights ranged from 2.0 to 2.8 feet due to differences in vessel operations, water depth, and river width along the project reach and wake periods were on the order of 3 to 4 seconds. The maximum wake height of 2.8 feet is taken as the design wake height from commercial vessels.

Commented [RC12]: Out of these vessel types, fireboats were not described in the second paragraph of this section (above). As a result, it is unclear which class of vessel they are representing. We suggest more clearly matching these design vessels with the vessel types noted in the previous paragraphs.

Additionally, the model was described as applying to all commercial vessels (except jet boats, which would be covered below). Tugboats, tows, cargo ships, and tankers are not presented in the table, but were listed as commercial vessels. We suggest explaining why all vessels were not included or explaining which vessels these design vessels are intended to represent.

Estimates of Wakes from Excursion Vessels (Jetboats)
The Weggel-Sorensen model (Weggel and Sorensen 1986) for evaluating ship wakes tends to over predict wakes created by faster moving recreation vessels. Therefore, a different methodology was used to estimate wakes produced by the excursion jet boats that operate in the Willamette River and throughout the Site in the summer season.

Many recent studies have addressed estimates of waves generated by different recreational ships, including numerous research studies by Maritime and Coastal Agency (MCA). Their most recent study included evaluation of wakes created by fast moving ferries (catamarans and mono-hull vessels) in water depths up to 20 meters (MCA 2009). The vessels and vessel operating conditions evaluated in this study are very similar to the jet boat operation within the Site. Therefore, the methodology developed by the MCA in the referenced report was used to estimate wakes created by the jet boat operations.

Estimates of waves generated by high-speed excursion boats, such as the Portland Spirit Outrageous Jetboat, were performed for two conditions: 1) jet boat traveling along the center line of the navigation channel, considered the most representative condition, and ~~(~~2) jet boat traveling half-way between the channel centerline and the bank, considered a rare operating condition~~s~~. The results of these calculations are presented in **Table C-7**.

Table C-7. Wake Heights Estimated for Excursion Jet Boats in Each SDU

SDU	REPRESENTATIVE CASE (Traveling at Center Line of Channel)				WORST CASE (Traveling 1/2 way between Center Line of Channel and Bank)			
	Water Depth (ft)	Distance from Sailing Line (ft)	Critical/ Supercritical	H (ft)	Water Depth (ft)	Distance from Sailing Line (ft)	Critical/ Supercritical	H (ft)
RM2E	49	1000	Supercritical	2.0	44	750	Supercritical	2.4
	58	1000	Supercritical	2.0	53	750	Supercritical	2.4
RM3.5E	44	900	Supercritical	2.2	44	650	Supercritical	2.6
	53	900	Supercritical	2.2	53	650	Supercritical	2.6
RM3.9W	44	900	Supercritical	2.2	49	650	Supercritical	2.6
	53	900	Supercritical	2.2	58	650	Supercritical	2.6
RM4.5E	69	750	Supercritical	2.4	59	500	Supercritical	2.9
	78	750	Supercritical	2.4	68	500	Supercritical	2.9
RM5W	44	550	Supercritical	2.8	49	350	Supercritical	3.5

SDU	REPRESENTATIVE CASE (Traveling at Center Line of Channel)				WORST CASE (Traveling 1/2 way between Center Line of Channel and Bank)			
	Water Depth (ft)	Distance from Sailing Line (ft)	Critical/ Supercritical	H (ft)	Water Depth (ft)	Distance from Sailing Line (ft)	Critical/ Supercritical	H (ft)
	53	550	Supercritical	2.8	58	350	Supercritical	3.5
RM5.5E, RM6W	49	500	Supercritical	2.9	44	333	Supercritical	3.6
	58	500	Supercritical	2.9	53	333	Supercritical	3.6
RM5.5E	59	625	Supercritical	2.6	49	500	Supercritical	2.9
	68	625	Supercritical	2.6	58	500	Supercritical	2.9
RM6.5E	49	750	Supercritical	2.4	49	625	Supercritical	2.6
	58	750	Supercritical	2.4	58	625	Supercritical	2.6
RM7W	44	500	Supercritical	2.9	39	375	Supercritical	3.4
	53	500	Supercritical	2.9	48	375	Supercritical	3.4
RM6.5E, RM7W	49	900	Supercritical	2.2	54	600	Supercritical	2.7
	58	900	Supercritical	2.2	63	600	Supercritical	2.7
SwanIs	44	600	Supercritical	2.7	39	500	Supercritical	2.9
	53	600	Supercritical	2.7	48	500	Supercritical	2.9
RM9W	39	1100	Supercritical	2.0	29	600	Supercritical	2.7
	48	1100	Supercritical	2.0	38	600	Supercritical	2.7
RM9W	39	750	Supercritical	2.4	25	400	Supercritical	3.3
	48	750	Supercritical	2.4	33	400	Supercritical	3.3
RM9W	35	900	Supercritical	2.2	44	375	Supercritical	3.4
	43	900	Supercritical	2.2	53	375	Supercritical	3.4
RM9E*, RM10E*	Ship travel at no-wake speed							
RM10W*	54	1000	Supercritical	2.0	34	500	Supercritical	2.9
	63	1000	Supercritical	2.0	43	500	Supercritical	2.9
RM11E	41	700	Supercritical	2.5	42	400	Supercritical	3.3
	50	700	Supercritical	2.5	50	400	Supercritical	3.3
RM11W*	44	700	Supercritical	2.5	44	400	Supercritical	3.3
	53	700	Supercritical	2.5	53	400	Supercritical	3.3

Notes:
* These are not official SDUs but represent approximate locations at the Site.

Wake heights range from 2.0 feet to 2.9 feet for the representative condition for jet boats and from 2.4 feet to 3.6 feet for the rare condition. The wake period of 4.0 seconds estimated for commercial vessels, is assumed to be the same for the jet boat excursion vessels to be conservative.

Findings

The analysis shows that erosion caused by wind and wake generated waves is likely limited to areas of the Site along the shoreline above 0 feet NAVD88. Within this zone, there is an area of likely heavier wave/wake action from 6 to 13 feet NAVD88 and area of likely less forceful wave/wake action from 0 to 6 feet NAVD88. Wave erosion effects above 13 feet NAVD88 were not evaluated because they were above the initially established Site boundary at the time the study was conducted.

C2.4.2 Shear Stress on Bottom Sediments

The GIS layer used to identify areas where the shear stress of a 2-year flow event exceeds the critical shear stress of the bedded sediment was developed using results from LWG's hydrodynamic model and sediment transport model. The 2-year return interval was considered reasonable because it delineates areas that are routinely affected by a flow event (occurs every 2 years) rather than areas that rarely (for example, every 100 years) experience flows that exceed the shear stress of the bedded sediment.

The hydrodynamic model is used to simulate temporal and spatial changes in water depth, current velocity, and bed shear stress. The sediment transport model inputs were used to determine critical bed shear stress. Erosive areas were defined as areas where the shear stress exceeded the critical bed shear stress for the 2-year recurrence flow event.

C2.4.2.1 Hydrodynamic Model – Shear Stress

The hydrodynamic model that was utilized in the study is the Environmental Fluid Dynamics Code (EFDC) model, which is supported by EPA. For this study, the two-dimensional (2D), depth-averaged hydrodynamic model within EFDC was used.

The hydrodynamic model requires specification of the following time-variable boundary conditions: 1) inflow at upstream boundary in the lower Willamette River; 2) inflow at upstream boundary in the Columbia River; 3) water surface elevation at downstream boundary in the Columbia River; and 4) water surface elevation at downstream boundary of the Multnomah Channel, this information is presented on **Figure C-2**. Daily-average flow rate data collected at the USGS Portland gauging station were used to specify the inflow at the upstream boundary in the lower Willamette River for the calibration and long-term simulations. Inflows at the upstream boundary during high-flow events were specified based on the results of a flood frequency analysis. A Log-Pearson Type 3 flood frequency analysis (Helsel and Hirsch 2002) of peak flow rate data from the 36-year historical record was conducted.

A summary of the estimated flow rates for high-flow events is presented in **Table C-8**. For comparison, the annual average flow rate is 33,200 cfs.

Table C-8. Estimated Lower Willamette River Flow Rates for High-flow Events

Flood Return Period (Years)	Flow Rate (cfs)
2	156,000
10	252,000
25	297,000
50	329,000
100	360,000
500	428,000

Notes:
cfs = cubic feet per second

Calibration of the hydrodynamic model was achieved using data collected with an Acoustic Doppler Current Profiler (ADCP) in the main channel of the lower Willamette River between River Mile (RM) 1 and 11. The ADCP data consisted of measurements of water depth and depth-averaged current velocity (magnitude and direction) during three different periods between 2002 and 2004. A summary of the three ADCP deployment periods is provided in **Table C-9**. Two of the survey periods in 2002 and 2003 were conducted approximately at or above the mean flow rate (26,000 to 66,000 cfs). The survey conducted in January 2004 was conducted during an approximate 2-year flood event.

Table C-9. ADCP Data Collection Summary

Survey Date	Lower Willamette River Flow Rate (cfs)	Survey Region	Number of Transects
April 19, 2002	66,000	RM 1 – 11	16
May 13, 2003	26,000	RM 2.5 – 4	4
January 31, 2004	139,000	RM 1 – 11	16

Notes:
cfs = cubic feet per second
RM = river mile

The model parameter that was adjusted to achieve the optimum agreement between predicted and observed water depth and current velocity was the effective bed roughness (Z_0) in the hydrodynamic model, which represents the total roughness due to form drag and skin friction. Generally, Z_0 ranges from about 0.1 to 10 centimeters (cm). A value of 1 cm for effective bed roughness produced the best agreement between observed and predicted water depth and depth-averaged current velocity during the calibration period.

Erosion rate is dependent on bed shear stress, which is calculated using current velocity predicted by the hydrodynamic model. The bed shear stress calculated within the hydrodynamic model is total bed shear stress, which represents the total drag on the water column by the sediment bed. The total bed shear stress (τ_{tot}) is the sum of shear stresses associated with skin friction (τ_{sf}) and form drag (τ_{fd}):

$$\tau_{tot} = \tau_{sf} + \tau_{fd} \qquad \text{Equation 1}$$

Skin friction represents the shear stress generated by sediment particles (i.e., small-scale physical features), whereas form drag corresponds to the drag generated by bedforms (e.g., ripples, dunes) and other large-scale physical features. When simulating erosion, skin friction is considered the dominant component of the bed shear stress for most applications. Thus, it is a reasonable approximation, and a standard approach, to use the skin friction component and neglect form drag for calculating bed shear stress for sediment transport simulations. This approach is consistent with accepted sediment transport theory (Parker 2004). Skin friction shear stress is calculated using the quadratic stress law:

$$\tau_{sf} = \rho_w \times C_f \times U^2 \qquad \text{Equation 2}$$

Where:

ρ_w = the density of water
C_f = the bottom friction coefficient
U = the depth-averaged current velocity.

The bottom friction coefficient is determined using (Parker 2004):

Equation 3

Where:

z_{ref} = a reference height above the sediment bed
k_s = the effective bed roughness
κ = von Karman's constant (0.4).

The reference height (z_{ref}) is spatially and temporally variable because it is equal to half of the water depth. Thus, the reference height properly incorporates temporal and spatial variations in water depth into the calculation of the bottom friction coefficient. The effective bed roughness is assumed to be proportional to the D_{90} of the surface sediment layer (Parker 2004; Wright and Parker 2004):

Commented [RC13]: This variable does not appear to be defined. We suggest defining it.

As such, it is difficult to evaluate whether or not this assumption is valid and what impact it would have. We suggest including some additional text to clarify these points. For example, for the two studies cited, did they study a similar river or explain why that is a reasonable assumption?

$$k_s = 2D_{90} \qquad \text{Equation 4}$$

Grain size distribution data were used to specify D_{90} values for the surface layer of lower Willamette River sediments. The spatial variability of D_{90} in the lower Willamette River was evaluated, accounting for potential spatial variation of D_{90} in the model produces qualitatively correct results (i.e., skin friction increases as bed roughness increases).

The validity of the above approach for calculating the bottom friction coefficient is evaluated as follows. Bottom friction coefficients were calculated for the lower

Willamette River, using representative D_{90} values in the cohesive and non-cohesive bed areas over a range of water depths (see **Table C-10**). The range of bottom friction coefficient values in **Table C-10** is consistent with expected values for cohesive beds (van Rijn 1993). This approach provides an objective method for estimating the effective bed roughness, which will decrease the uncertainty associated with subjective estimates of roughness.

Table C-10. Bottom Friction Coefficient Values for a Range of Water Depths

Water Depth (m)	Bottom Friction Coefficient: Cohesive Bed (D90 = 280 μm)	Bottom Friction Coefficient: Non-Cohesive Bed (D90 = 1,480 μm)
1	0.0016	0.0024
2	0.0014	0.0020
3	0.0013	0.0018
4	0.0012	0.0017

Notes:
μm = micrometer
m = meter

For use in sediment transport formulations, a demonstrated accurate equation for bed shear velocity (u*) is defined as (van Rijn 1993):

$$u^{*} = (\tau_{sf} / \rho_{w})^{0.5} \qquad \text{Equation 5}$$

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Current velocity in turbulent flow, which exists in the lower Willamette River for all flow and tidal conditions, is the sum of two components: time-averaged mean velocity and turbulent fluctuations about the mean value. The bed shear velocity (u*) corresponds to the turbulent-fluctuation component of the current velocity. Thus, the skin friction shear stress is driven by the turbulent fluctuations in the flow, which are randomly variable with time.

C2.4.2.2 Sediment Transport Model Input –Bed Properties

Sediment transport model inputs for sediment bed properties were used to determine critical bed shear stress across the Site. Bed properties range from bulk bed characteristics such as dry density and grain size distribution to erosion rates.

The sediment bed in the lower Willamette River was separated into three distinct types:

1) cohesive (i.e., muddy bed composed of a mixture of clay, silt, sand, and organic matter)
2) non-cohesive (i.e., sandy bed composed of sand and gravel, with small amounts of clay and silt)
3) hard bottom (i.e., no erosion or deposition)

Delineation of the sediment bed into cohesive, non-cohesive and hard bottom areas was accomplished using grain size distribution data from sediment cores collected during the GeoSea and Round 2 field studies during 2000 and 2004, respectively (GeoSea 2001; Integral 2005a, 2005b, 2006). Grain size distribution data were available at a total of 1,187 locations at the Site (see **Figures C-3a and C-3b**). Sediment cores were classified as cohesive using the following criteria: 1) median particle diameter (D_{50}) less than 250 micrometers (µm); and 2) clay/silt content greater than 15 percent (Ziegler and Nisbet 1994). The sediment bed was assumed to be ~~was~~ hard bottom in the following areas~~vin~~ upstream of RM 12.9 in the lower Willamette River, Multnomah Channel, and the Columbia River. The bed map for the Site is shown on **Figure C-4**. About 81 percent of the bed area between RMs 2 and 11 is cohesive.

The following bed property inputs within the lower Willamette River were determined for use in the sediment transport model:

1) dry (bulk) density
2) initial sediment bed composition (i.e., relative amounts of sediment sizes)
3) median particle diameter (D_{50})
4) effective bed roughness (which is proportional to D_{90})
5) erosion rate properties in cohesive bed areas

The dry density of the bed was assumed to be spatially variable within the lower Willamette River, with different values in the cohesive and non-cohesive bed areas. For cohesive bed areas, the dry density has a value of 0.72 grams per cubic centimeter~~s~~ (g/cm^3), which corresponds to the average value of 596 samples. For non-cohesive bed areas, the dry density has a value of 1.2 g/cm^3, which corresponds to the average value of 162 samples. Dry density is assumed to be horizontally and vertically constant within all areas of a particular bed type.

Spatial distributions of D_{50} and D_{90} values were developed from the grain size distribution data collected at 1,187 locations at the Site (**Figure C-5**). Spatial distributions of bed composition were specified as initial conditions for the sediment transport model using the grain size distribution data (**Figures C-6a** through **C-6b**). As a reference, **Table C-11** presents the average values of D_{50}, D_{90}, and composition of the bed for cohesive and non-cohesive areas.

Table C-11. Average Values for Bed Properties Initial Conditions

Bed Type	D_{50} (µm)	D_{90} (µm)	Class 1 Content (%)	Class 2 Content (%)	Class 3 Content (%)	Class 4 Content (%)
Cohesive	50	280	64	26	9	1
Non-Cohesive	510	1,480	13	14	64	9

Notes:
µm = micrometer
Class 1 = Clay and silt with particle diameters less than 62 µm
Class 2 = Fine sand (62 to 250 µm)
Class 3 = Medium and coarse sand (250 to 2,000 µm)
Class 4 = Gravel (greater than 2,000 µm)

A Sedflume study was conducted during 2006 to obtain data on the erosion properties of lower Willamette River sediments. Cores were collected from 19 locations (**Figure C-7**). Details of the field study, including core collection and processing, are described in Sea Engineering (2006). Erosion rates as a function of depth in the bed and applied shear stress were measured over the top 30 cm of each core using Sedflume. Sediment samples were also obtained at 5-cm intervals from each core and analyzed for bulk (wet) density and grain size distribution.

Erosion rate data obtained from Sedflume testing were analyzed to develop an understanding of the erosion properties of lower Willamette River sediments in cohesive bed areas. The goal of this analysis was to develop a functional relationship between the gross erosion rate (E_{gross}) and bed shear stress. The site-specific parameters in the E_{gross} equation below were determined using the erosion rate data collected during the field study. Four of the 19 Sedflume cores (SF-2, SF-6, SF-7, SF-18) were determined to consist of non-cohesive (i.e., sandy) sediment and those cores were not included in the analysis as Sedflume erosion rate data are only applicable to cohesive bed sediment.

$$E_{gross} = A\,\tau_{sf}^{n} \quad \text{for } \tau_{sf} > \tau_{cr} \qquad \text{Equation 6}$$
$$= 0 \quad \text{for } \tau_{sf} \le \tau_{cr}$$

Where:

E_{gross} = gross erosion rate (centimeters per second [cm/s])
τ_{sf} = skin friction shear stress (Pascal [Pa])
τ_{cr} = critical shear stress (Pa), which is the shear stress at which a small, but measurable, rate of erosion occurs (generally less than 2 millimeters per hour).

Commented [RC14]: In addition to stating the name of the unit, we suggest you consider expressing this in terms of basic SI units (kg/ms^2).

The erosion parameters, A (proportionality constant) and n (exponent), are site-specific and may be spatially variable, both horizontally and vertically.

The erosion rate properties of the 15 cores were analyzed using the following procedure. Each core was divided into five depth intervals of 5 cm each between 0 and 25 cm. These depth intervals were chosen because the shear stress series used in the Sedflume tests, where shear stress was increased from low to high values, were cycled over approximately 5 cm thick layers. The erosion rate data within each layer of a particular core were analyzed through application of a log-linear regression analysis between erosion rate and shear stress. The log-linear regression analysis produced values of A and n for each layer in a particular core. The results of this analysis for the Sedflume cores with cohesive sediment are presented in **Figures C-8** through **C-22**. The critical shear stress for each 5 cm layer was calculated as:

$$\tau_{cr} = (E_{cr} / A)^{1/n} \qquad \text{Equation 7}$$

Where:

E_{cr} = 0.0001 cm/s

The erosion rate parameters (i.e., A, n, τ_{cr}) for each core within the five depth intervals are listed in **Tables C-12** through **C-16**. Note that the values of A and n in these tables correspond to units of cm/s for E_{gross} and pascal (Pa) for bed shear stress. The correlation coefficient (R^2) values presented in the tables are from the log-linear regression analysis, with perfect correlation corresponding to an R^2 value of one.

Table C-12. Erosion Rate Parameters for 0 to 5 cm Layer

Sediment Core ID	River Mile Location	Proportionality Constant: A	Exponent: n	Correlation Coefficient (R^2)	Critical Shear Stress (Pa)
SF-1	2.4	0.00113	2.4	0.97	0.36
SF-3	3.7	0.00504	1.6	0.96	0.09
SF-4	4.0	0.00244	2.3	0.99	0.25
SF-5	4.8	0.00137	2.0	0.94	0.27
SF-8	6.1	0.00473	2.7	0.98	0.24
SF-9	6.4	0.00081	2.0	0.80	0.35
SF-10	6.8	0.00110	2.25	0.95	0.33
SF-11	6.9	0.00025	3.1	0.98	0.73
SF-12	7.6	0.00430	1.6	0.92	0.10
SF-13	8.0	0.00218	1.3	0.76	0.10
SF-14	8.3	0.00140	1.3	0.76	0.14
SF-15	8.6	0.00546	2.1	0.96	0.15
SF-16	9.3	0.00065	2.6	0.90	0.49
SF-17	10.0	0.00061	2.9	0.96	0.54
SF-19	10.4	0.00115	2.3	0.95	0.34

Table C-13. Erosion Rate Parameters for 5 to 10 cm Layer

Sediment Core ID	River Mile Location	Proportionality Constant: A	Exponent: n	Correlation Coefficient (R^2)	Critical Shear Stress (Pa)
SF-1	2.4	0.00106	2.8	0.99	0.43
SF-3	3.7	0.00056	4.6	0.99	0.69
SF-4	4.0	0.00043	3.7	0.97	0.67
SF-5	4.8	0.00014	3.2	0.96	0.89
SF-8	6.1	0.00151	2.1	0.96	0.28
SF-9	6.4	0.00015	3.1	0.99	0.86
SF-10	6.8	0.00036	3.1	0.99	0.66
SF-11	6.9	0.00002	4.4	0.97	1.33
SF-12	7.6	0.00054	2.4	0.99	0.49
SF-13	8.0	0.00115	2.6	0.95	0.38
SF-14	8.3	0.00014	1.9	0.83	0.83
SF-15	8.6	0.00117	2.1	0.96	0.32
SF-16	9.3	0.00306	2.0	0.93	0.18
SF-17	10.0	0.00047	3.0	0.98	0.59
SF-19	10.4	0.00120	2.2	0.60	0.32

Table C-14. Erosion Rate Parameters for 10 to 15 cm Layer

Sediment Core ID	River Mile Location	Proportionality Constant: A	Exponent: n	Correlation Coefficient (R^2)	Critical Shear Stress (Pa)
SF-1	2.4	0.00048	3.9	0.98	0.67
SF-3	3.7	0.00608	2.8	0.98	0.23
SF-4	4.0	0.00034	2.8	0.99	0.64
SF-5	4.8	0.00026	2.6	0.99	0.68
SF-8	6.1	N/A	N/A	N/A	N/A
SF-9	6.4	0.00039	2.3	0.93	0.56
SF-10	6.8	0.00008	3.0	0.95	1.08
SF-11	6.9	0.00358	1.7	0.89	0.12
SF-12	7.6	0.00132	1.8	0.99	0.23
SF-13	8.0	0.00030	2.7	0.90	0.66
SF-14	8.3	0.00003	2.8	0.94	1.47
SF-15	8.6	0.00039	3.3	0.97	0.66
SF-16	9.3	0.00163	2.8	0.94	0.37
SF-17	10.0	0.00040	3.0	0.93	0.63
SF-19	10.4	0.00088	2.9	0.84	0.47

Table C-15. Erosion Rate Parameters for 15 to 20 cm Layer

Sediment Core ID	River Mile Location	Proportionality Constant: A	Exponent: n	Correlation Coefficient (R^2)	Critical Shear Stress (Pa)
SF-1	2.4	0.00097	2.4	0.99	0.39
SF-3	3.7	0.00706	2.8	0.96	0.22
SF-4	4.0	0.00096	2.4	0.95	0.39
SF-5	4.8	0.00082	2.4	0.99	0.42
SF-8	6.1	N/A	N/A	N/A	N/A
SF-9	6.4	0.00027	2.5	0.92	0.66
SF-10	6.8	0.00004	3.1	0.99	1.30
SF-11	6.9	0.00358	1.7	0.89	0.11
SF-12	7.6	0.00090	2.8	0.99	0.45
SF-13	8.0	0.00025	3.1	0.95	0.74
SF-14	8.3	0.00003	2.7	0.88	1.54
SF-15	8.6	0.00002	4.6	0.99	1.41
SF-16	9.3	0.01233	1.1	0.86	0.02
SF-17	10.0	0.00077	2.2	0.77	0.40
SF-19	10.4	0.00409	1.8	0.82	0.13

Table C-16. Erosion Rate Parameters for 20 to 25 cm Layer

Sediment Core ID	River Mile Location	Proportionality Constant: A	Exponent: n	Correlation Coefficient (R^2)	Critical Shear Stress (Pa)
SF-1	2.4	0.00049	2.8	0.96	0.56
SF-3	3.7	0.00825	2.7	0.98	0.20
SF-4	4.0	0.00056	2.9	0.95	0.55
SF-5	4.8	0.00026	3.0	0.95	0.72
SF-8	6.1	N/A	N/A	N/A	N/A
SF-9	6.4	0.00004	3.2	0.99	1.33
SF-10	6.8	0.00006	2.7	0.97	1.18
SF-11	6.9	N/A	N/A	N/A	N/A
SF-12	7.6	0.00037	3.5	0.99	0.69
SF-13	8.0	0.00011	3.7	0.84	0.97
SF-14	8.3	0.00003	2.8	0.97	1.42
SF-15	8.6	0.00006	3.1	0.99	1.13
SF-16	9.3	0.01254	1.3	0.99	0.02
SF-17	10.0	0.00003	3.9	0.99	1.36
SF-19	10.4	0.00239	2.6	0.72	0.30

Spatial variation, both horizontal and vertical, in the erodibility of sediments in the lower Willamette River cohesive bed areas was evaluated as follows. The first step was to calculate average values of the A and n parameters in Equation 6 for each of the five depth intervals. For a log-linear relationship (such as Equation 6), the average exponent (n_{ave}) value for a depth interval is the arithmetic average of the n values for the cores within the interval. The average proportionality constant (A_{ave}) is determined by calculating the log-average value:

$$\log(A_{ave}) = (1/K) \sum \log(A_k) \qquad \text{Equation 7}$$

where K is equal to the number of cores. Using this approach, the average erosion parameters for the five layers in the bed model are listed in **Table C-17**.

Table C-17. Vertical Variation in Average Erosion Rate Parameters

Depth Interval	Average Proportionality Constant: A_{ave}	Average Exponent: n_{ave}	Critical Shear Stress (Pa)
Layer 1: 0 – 5 cm	0.00155	2.2	0.28
Layer 2: 5 – 10 cm	0.00048	2.9	0.58
Layer 3: 10 – 15 cm	0.00052	2.7	0.55
Layer 4: 15 – 20 cm	0.00062	2.6	0.49
Layer 5: 20 – 25 cm	0.00032	2.9	0.66

Vertical variation in the average erosion rate properties for the five depth intervals was quantified using the following procedure. First, calculate the average value of gross erosion rate for depth interval i (${}^{ave}E_{gross,i}$, where i ranges from 1 to 5):

$$^{ave}E_{gross,i} = 1/N \ \Sigma \ A_{ave,i} \ \tau^{n,ave,i} \qquad \text{Equation 8}$$

where the summation is over the bed shear stress range of 0.05 to 3 Pa in increments of 0.05 Pa, so N is equal to 60. Values of $A_{ave,i}$ and $n_{ave,i}$ for depth interval i are given in **Table C-17**. Using the values of ${}^{ave}E_{gross,i}$ for the five depth intervals, the average erosion rate ratios for depth interval i ($R_{ave,i}$) was calculated using:

$$R_{ave,i} = {}^{ave}E_{gross,i} / {}^{ave}E_{gross,1} \qquad \text{Equation 9}$$

where i ranges from 1 through 5. Thus, $R_{ave,i}$ represents the ratio of the erodibility of depth interval i to the average erodibility of depth interval 1 (i.e., 0 to 5 cm layer); $R_{ave,1}$ is equal to one. The vertical variation in $R_{ave,i}$ is shown on **Figure C-23**. These results show that the average erodibility of lower Willamette River sediment in cohesive bed areas tends to decrease with increasing depth in the bed, which is a typical characteristic of a cohesive sediment bed and is primarily due to increasing consolidation with increasing depth. Erodibility of the 20 to 25 cm layer is about four times less than the erodibility of the 0 to 5 cm layer.

A similar approach was used to quantify spatial differences in bed erodibility of the surface layer (i.e., 0 to 5 cm layer) within the horizontal plane in the lower Willamette River. The average gross erosion rate for layer 1 (0 to 5 cm layer) in core k was calculated as follows:

$$^{ave}E_{gross,1,k} = 1/N \ \Sigma \ A_{1,k} \ \tau^{n,1,k} \qquad \text{Equation 10}$$

where the summation is over the bed shear stress range of 0.05 to 3 Pa in increments of 0.05 Pa, so N is equal to 60. Values of $A_{1,k}$ and $n_{1,k}$ for layer 1 in core k are given in **Table C-12**.

Sedflume data from 15 cores are not sufficient to use standard interpolation methods to develop a reliable horizontal distribution of erosion properties. No correlation was found between erosion properties and measured bed properties (i.e., dry density, D_{50}, D_{90}, silt/clay content). Thus, developing a credible spatial distribution of erosion parameters in the horizontal plane is problematic. Therefore, it was assumed that the average erosion rate parameters (A_{ave} and n_{ave} as listed in **Table C-17**) for a given depth interval are spatially constant in the horizontal plane within cohesive bed areas. By assuming that the erosion parameters are spatially constant in the horizontal plane, the erosion parameters only vary in the vertical direction.

C2.5 SHALLOW WATER DEPTH

Shallow water zones were identified using January 2009 bathymetry data and identifying areas at or greater than 4 feet NAVD88. The shallow water criterion of 4 feet NAVD88 was based on an assumed cap thickness of 3 feet and a mean lower low water (MLLW) elevation of 7 feet NAVD88. This allows for construction of a 3-foot cap that remains submerged at the MLLW.

C3.0 SEDIMENT BED CHARACTERISTICS

The source of information used to designate sediment slope areas is provided below.

C3.1 SEDIMENT SLOPE

The January 2009 bathymetry data was used to identify sediment slopes within the Portland Harbor Superfund Site. Slopes less than 15 percent, between 15 and 30 percent, and greater than 30 percent were delineated based on the technology assignment criteria discussed in FS Section 3.6.9.

C4.0 ANTHROPOGENIC INFLUENCES

The sources of information used to identify structures and pilings, delineate moderate to high debris areas, and identify propwash areas are provided below.

C4.1 STRUCTURES AND PILINGS

The GIS layer used to identify structures and pilings at the Site was created using two layers developed by the LWG. One layer identified docks and other structures, a second layer described the approximate distribution of structures and debris in the river channel and along both banks of the river based on a high resolution sidescan sonar survey in 2008. The sidescan sonar survey area extended from RM 1 to RM 12.2, and included the half mile uppermost segment of the Multnomah Channel. A total of 7,257 discrete targets from the area surveyed were identified. A detailed presentation of targets and their locations is provided in the *Lower Willamette River Sidescan Sonar Data Report* (Anchor QEA 2009).

Approximately two thirds of the targets identified were clearly man-made objects (piers, pilings, dolphins, and structures) placed in the river for navigational, operational, or engineering purposes. Approximately 25 percent of the remaining material was broadly classified as debris. Logs accounted for approximately 5 percent of the targets. Other geologic and cultural features observed using sidescan sonar included the occurrence of gravel, depressions, anchor drags, and dredge artifacts. Targets identified as debris, logs, or other miscellaneous features were removed from the GIS layer. All remaining targets identified as structures in the queried file were buffered with a five foot radius and then combined with the docks and structures GIS layer. The combined layer was then converted to a raster file for analysis purposes.

C4.2 DEBRIS

The GIS debris layer initially came from the same high resolution sidescan sonar survey described in Section E5.1 above. As discussed, approximately 25 percent of the targets identified during the sidescan sonar survey were broadly classified as debris. Debris was commonly found along the margins of dock structures, a pattern that is consistent with vessel activity patterns. The logs that accounted for approximately 5 percent of the targets were often associated with areas that are or were log booming areas.

The original GIS layer provided by LWG from the survey was modified to only include targets identified as debris, logs, or unclassified. Structures, pilings, and dolphins were removed from the debris layer. The new layer was then converted into a vector file for analysis purposes using a method called Point Density, which calculates the density of point features around each raster cell. The raster file consists of 10 foot by 10 foot cells. A neighborhood was defined as a circle with a 50 foot radius, and was based around each raster cell center. Then the number of points that fell within the circle were totaled and divided by the area of the neighborhood. The area units were set to acres, so the calculated density for each cell was multiplied by the appropriate factor and then written to the output raster. The resulting raster was reclassified so that any cell with a value less than or equal to 40 was set to 0. Any cell with a value greater than 40 was set to 1 and identified as containing moderate to heavy debris.

C4.3 PROPWASH

The GIS layer used to define propwash areas was provide by the LWG on January 22, 2014. The LWG conducted modeling to determine potential surface sediment mixing and scour depths due to propwash forces based on the vessels and operating parameters determined through the site use survey discussed in Section E2.1.

Propwash disturbance depths were evaluated using the following specific methods:

- Dücker and Miller (1996)
- Hamill (1988)

The Dücker and Miller (1996) method predicts the disturbance depth based on the bed sediment grain size, jet velocity at the bed, rudder angle, and distance between the propeller and bed. The Hamill (1988) method predicts disturbance depth based on the clearance of the propeller tip above the bed, the diameter of the propeller, jet velocity at the bed, sediment grain size, and time of exposure to the propeller wash (a time rate of scour). For this method, a time of exposure of 120 seconds (2 minutes) was assumed. This method is sensitive to this assumption, but 2 minutes was selected as a reasonably conservative estimate given that these propwash effects are usually transitory to any particular location and of much shorter duration even in the case of most docking situations.

Commented [RC15]: We suggest quantifying or describing the sensitivity in more detail.

Both methods were used to evaluate propwash based on vessel parameters presented in **Table C-18** and the range of vessels estimated to operate across a range of SDUs. These input parameters were selected to span a range of Site conditions that are likely representative of propwash forces and conditions for the Site as a whole, and are representative of current vessel operations in the SDUs based on the site use survey, with the exception of SDU RM2E. The Evraz Oregon Steel Mills dock is located in this SDU, and the docks in SDU RM3.5E and SDU7W, are currently inactive. Thus, they were evaluated assuming vessels that are representative of those that may use the area in the future.

Table C-18. Vessel Data

Vessel Class	Propeller Shaft Depth (ft)	Possible or Potential SDUs Where Vessels Likely Operate	Propeller Diameter (ft)	Vessel Horse power (HP)	Maximum Reasonable HP Applied[a] (%)
Large tug	13	RM2E, RM3.5E, RM7W, SwanIs,	8 (twin)	3,300	80
Small tug	9	RM2E, RM3.9W, RM5W, RM6W,	6 (twin)	2,000	80
Large ocean-going vessel	30 to 31	RM2E, RM11E	18	20,000	30
Medium ocean-going vessel	23	SwanIs	14	20,000	30
International cargo ship	28 to 31	RM3.5E, RM3.9W	18	20,000	30
Ocean-going hopper dredge	20	RM6W	10	15,000	30
Fishing vessel	5	SwanIs	3	250	80
Pleasure craft	5	RM5.5E	3	250	90

Notes:
[a] Maximum horsepower estimated based on reasonable maximum under typical operating conditions.

Commented [RC16]: Although the footnote describes this term in some way, we suggest explaining how the HP were actually established as "reasonable". For instance, were these values based on discussions with captains?

Table C-19. Stable Sediment Size under Maximum Velocity Scenario and Reasonable Conservative Case Assumptions

SDU[a]	Design Vessel	Minimum Water Depth in Areas of Operation (ft)	C_3 (frequency coefficient)[b]	Max V_b (fps)	Stable Sediment Size D_{50} (in)	Sediment Description
RM2E	Large tug	30	0.7	2.8	3.5	cobbles
RM2E	Lg. ocean-going vessel	40	0.7	10.2	48.0	riprap
RM2E	Large tug	35	0.6	2.1	2.9	coarse gravel
RM2E	Small tug	40	0.5	1.2	1.2	coarse gravel
RM3.5E	Large tug	25	0.7	3.9	7.1	cobbles
RM3.5E	Int'l cargo ship	40	0.7	8.5	33.3	riprap
RM3.9W	Int'l cargo ship	45	0.7	7.3	24.5	riprap
RM3.9W	Small tug	25	0.7	2.3	2.4	coarse gravel
RM3.9W	Small tug	25	0.6	2.3	3.2	cobbles

SDU[a]	Design Vessel	Minimum Water Depth in Areas of Operation (ft)	C_3 (frequency coefficient)[b]	Max V_b (fps)	Stable Sediment Size D_{50} (in)	Sediment Description
RM5W	Small tug	20	0.5	3.3	9.7	cobbles
RM5W	Small tug	30	0.6	1.7	1.9	coarse gravel
RM6W	Small tug	20	0.5	3.3	9.7	cobbles
RM6W	Small tug	25	0.7	2.3	2.4	coarse gravel
RM6W	Ocean-going hopper dredge	45	0.7	2.4	2.7	coarse gravel
RM6W	Small tug	25	0.5	2.3	4.6	cobbles
RM5.5E	Pleasure craft	10	0.5	3.8	12.8	riprap
RM7W	Large tug	30	0.5	2.8	6.9	cobbles
SwanIs	Large tug	25	0.7	3.9	7.1	cobbles
SwanIs	Md. ocean-going vessel	50	0.7	3.5	5.6	cobbles
SwanIs	Md. ocean-going vessel	30	0.6	13.4	>60	riprap
SwanIs	Fishing vessel	10	0.5	3.6	11.8	cobbles
RM9W	Small tug	20	0.5	3.3	9.7	cobbles
RM9W	Small tug	20	0.5	3.3	9.7	cobbles
RM9W	Large tug	35	0.7	2.1	2.1	coarse gravel
RM9W	Small tug	25	0.7	2.3	2.4	coarse gravel
RM9W	Small tug	25	0.6	2.3	3.2	cobbles
RM9W	Small tug	15	0.7	6.0	16.7	riprap
RM9E*	Md. ocean-going vessel	50	0.7	3.5	5.6	cobbles
RM10W*	Lg. ocean-going vessel	45	0.6	7.3	33.3	riprap
RM11E	Small tug	45	0.5	1.0	0.9	coarse gravel
RM11E	Lg. ocean-going vessel	50	0.6	5.4	18.1	riprap

Notes:
a Note that there is no vessel activity reported at SDU RM4.5E and RM6.5E.
b. The C3 parameter represents frequency of vessel operations. Infrequent = 0.7, moderate = 0.6, frequent = 0.5.
* These are not official SDUs but represent approximate locations at the Site.

The resulting disturbance depths from propwash forces across a range of potential Site conditions are summarized in **Table C-20**. In some instances the combination of parameters could not be used to resolve an exact disturbance depth using the Hamill method. In addition, estimates of greater than a 6-foot disturbance depth may be also beyond the range of parameters that this method can reasonably resolve, given that they differ significantly from the findings using the Dücker and Miller method.

Commented [RC17]: We suggest providing an example of the parameter(s) that cause a problem for this method.

Table C-20. Summary of Propwash Disturbance Depth Estimates Using Two Methods

SDU	Representative Vessel	Dücker and Miller Disturbance Depth (ft)	Hamill Disturbance Depth (ft)	Maximum Disturbance Depth (ft)
RM2E	Large tug	0.50	0.03	0.50
RM2E	Lg. ocean-going vessel	> 1*	> 1*	> 1*
RM2E	Large tug	0.34	0.01	0.34
RM2E	Small tug	0.13	< 0.01	0.13
RM3.5E	Large tug	0.68	0.12	0.68
RM3.5E	Int'l cargo ship	> 1*	> 1*	1.00
RM3.9W	Int'l cargo ship	> 1*	6.76	6.76
RM3.9W	Small tug	0.37	0.02	0.37
RM3.9W	Small tug	0.37	0.02	0.37
RM5W	Small tug	0.55	0.06	0.55
RM5W	Small tug	0.24	0.06	0.24
RM6W	Small tug	0.55	0.06	0.55
RM6W	Small tug	0.37	0.02	0.37
RM6W	Ocean-going hopper dredge	0.39	0.01	0.39
RM6W	Small tug	0.37	0.02	0.37
RM5.5E	Pleasure craft	0.66	0.24	0.66
RM7W	Large tug	0.50	0.03	0.50
SwanIs	Large tug	0.68	0.12	0.68
SwanIs	Md. ocean-going vessel	0.60	0.03	0.60
SwanIs	Md. ocean-going vessel	> 1*	> 1*	> 1*
SwanIs	Fishing vessel	0.63	0.21	0.63
RM9W	Small tug	0.55	0.06	0.55
RM9W	Small tug	0.55	0.06	0.55
RM9W	Large tug	0.34	0.01	0.34
RM9W	Small tug	0.37	0.02	0.37
RM9W	Small tug	0.37	0.02	0.37
RM9W	Small tug	> 1*	2.77	2.77
RM9E+	Md. ocean-going vessel	0.60	0.03	0.60
RM10W+	Lg. ocean-going vessel	> 1*	6.76	6.76
RM11E	Small tug	0.05	< 0.01	0.05
RM11E	Lg. ocean-going vessel	> 1*	0.38	> 1*

Note:
For some of the SDUs, several locations within the SDU were evaluated and these varying locations are shown above.
* For the Hamill and Dücker and Miller methods, an exact depth was not resolvable for the representative vessel parameters.
+ These are not official SDUs but represent locations at the Site.

The disturbance depth estimates indicate that maximum disturbance depths under most of the conditions applicable to the Site are less than 1 foot, even in heavier propwash areas, located in relatively shallower water areas of the navigation channel and near active docks. However, in specific areas and under specific conditions, greater depths of sediment disturbance might be expected to take place. This concept is supported by bathymetry information, which indicate that so-called “scour pits” may exist in and near some berthing areas, although this does not appear to occur everywhere that vessels travel or dock. Given the level of detail of vessel operations available for the FS, the specific situations where these greater disturbance depths are likely to occur is a design-level issue that will need to be resolved in SMA-specific remedial designs.

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Attachment C-1
Wake Analysis Figures

Commented [RC18]: Does this refer to the "Figures Attachment" or actual data tables and screenshots of the analysis? If the former, we suggest including "Figures" at the end of this phrase, as shown.